

## Chapter 7 Commentary

### FOUNDATION DESIGN REQUIREMENTS

#### 7.1 GENERAL

**7.1.1 Scope.** The minimum foundation design requirements that might be suitable when any consideration must be given to earthquake resistance are set forth in Chapter 7. It is difficult to separate foundation requirements for minimal earthquake resistance from the requirements for resisting normal vertical loads. In order to have a minimum base from which to start, this chapter assumes compliance with all basic requirements necessary to provide support for vertical loads and lateral loads other than earthquake. These basic requirements include, but are not limited to, provisions for the extent of investigation needed to establish criteria for fills, slope stability, expansive soils, allowable soil pressures, footings for specialized construction, drainage, settlement control, and pile requirements and capacities. Certain detailing requirements and the allowable stresses to be used are provided in other chapters of the *Provisions* as are the additional requirements to be used in more seismically active locations.

#### 7.2 GENERAL DESIGN REQUIREMENTS

**7.2.2 Soil capacities.** This section requires that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil. It is noted that the Appendix to Chapter 7 introduces into the *Provisions* ultimate strength design (USD) procedures for the geotechnical design of foundations. The *Commentary* Appendix to Chapter 7 provides additional guidance and discussion of the USD procedures.

**7.2.3 Foundation load-deformation characteristics.** The Appendix to Chapter 7 (*Provisions* and *Commentary* (Sec. A7.2.3) provides guidance on modeling load-deformation characteristics of the foundation-soil system (foundation stiffness). The guidance contained therein covers both linear and nonlinear analysis methods.

#### 7.3 SEISMIC DESIGN CATEGORY B

There are no special seismic provisions for the design of foundations for buildings assigned to Seismic Design Category B.

#### 7.4 SEISMIC DESIGN CATEGORY C

Extra precautions are required for the seismic design of foundations for buildings assigned to Seismic Design Category C.

**7.4.1 Investigation.** This section reviews procedures that are commonly used for evaluating potential site geologic hazards due to earthquakes, including slope instability, liquefaction, and surface fault rupture. Geologic hazards evaluations should be carried out by qualified geotechnical professionals and documented in a written report.

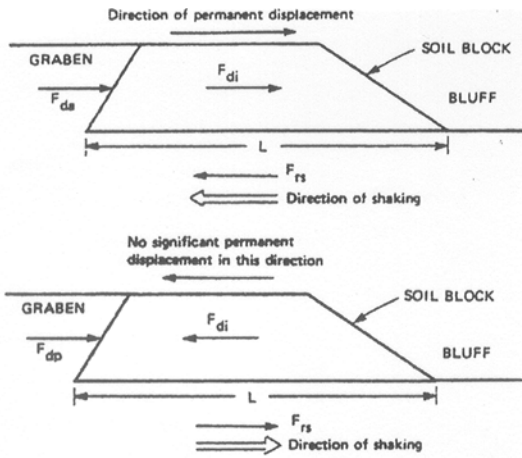
**Screening Evaluation.** Evaluation of geologic hazard may initially consist of a screening evaluation. If the screening evaluation clearly demonstrates that a hazard is not present, then more detailed evaluations, using procedures such as those described in the following sections, need not be conducted. Reference to the following publications are suggested for guidelines on screening evaluations: California Division of Mines and Geology (1997) – slope instability; Blake et al. (2002) and Stewart et al. (2003) – slope instability; Martin and Lew (1999) – liquefaction; U.S. Army Corps of Engineers (1998) – slope instability; liquefaction; surface fault rupture. More detailed evaluation procedures such as those described below should be used if a hazard cannot be screened out.

**Slope instability hazard.** The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using standard procedures.

For initial evaluation, the pseudostatic analysis may be used. (The deformational analysis described below, however, is now preferred.) In the pseudostatic analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the peak ground acceleration,  $a_{max}$  or  $S_{DS}/2.5$ . The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than one indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than one indicates that the slope will yield and slope deformation can be expected and a deformational analysis should be made using the techniques discussed below.

Deformational analyses resulting in estimates of slope displacement are now accepted practice. The most common analysis, termed a Newmark analysis (Newmark, 1965), uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration. The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. See Figure C7.4-1 for forces and equations used in analysis and Figure C7.4-2 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

Acceptable methods for the determination of displacements on many projects involve the use of charts that show displacements for different acceleration ratios, where the acceleration ratio is defined as the ratio of yield acceleration to peak ground acceleration. Various charts have been developed, including those by Franklin and Chang (1977), Makdisi and Seed (1978), Wong and Whitman (1982), Hynes and Franklin (1984), Martin and Qiu (1994); and Bray and Rathje (1998). The selection between the different charts should be made on the basis of the type of slope and the degree of conservatism necessary for the project. A number of the chart methods were developed for the estimation of displacements for dams, and therefore, may be more suitable for embankment designs. Recommendations on the use of such procedures for typical building construction are presented by Blake et al. (2002).



- $F_{da}$  = driving force due to active soil pressure
- $F_{di}$  = driving force due to earthquake inertia
- $F_{rs}$  = resisting force due to soil shear strength
- $F_{dp}$  = resisting force due to passive soil pressure
- $F_{di} = K_{max} W$

where  $K_{max}$  = maximum seismic coefficient and  $W$  = weight of soil block

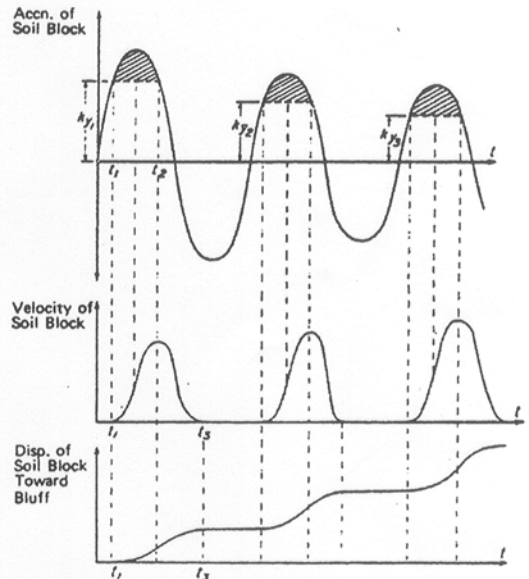
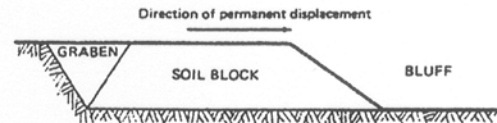
$$F_{rs} = S_u L$$

where  $S_u$  = average undrained shear strength of soil and  $L$  = length of soil block

Yield seismic coefficient:

$$K_y = \frac{F_{rs} - F_{da}}{W}$$

**Figure C7.4-1** Forces and equations used in analysis of translatory landslides for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985)



**Figure C7.4-2** Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966)

**Mitigation of slope instability hazard.** With respect to slope instability, three general mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement. Techniques to stabilize a site include reducing the driving forces by grading and drainage of slopes and increasing the resisting forces by subsurface drainage, buttresses, ground anchors, reaction piles or shafts, ground improvement using densification or soil mixing methods, or chemical treatment.

**Liquefaction hazard.** Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. Loss of bearing strength, differential settlement, and horizontal displacement due to lateral spreads have been the direct causes of damage. Examples of this damage can be found in reports from many of the more recent earthquakes in the United States, including the 1964 Alaska, the 1971 San Fernando, the 1989 Loma Prieta, the 1994 Northridge, the 2001 Nisqually, and the 2003 Denali earthquakes. Similar damage was reported after the 1964 Niigata, the 1994 Hyogoken-Nanbu (Kobe), the 1999 Taiwan, and the 1999 Turkey earthquakes. As earthquakes occur in the future, additional cases of liquefaction-related damage must be expected. Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and mitigating the hazard by designing to resist ground displacement or strength loss, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard.

**Evaluation of liquefaction hazard.** Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question to define a cyclic resistance ratio (CRR) and a cyclic stress ratio induced by the earthquake (CSR).

The following possible methods for calculating the factor of safety against liquefaction have been proposed and used to various extents:

1. **Empirical Methods**—The most widely used method in practice involves empirical procedures. These procedures rely on correlations between observed cases of liquefaction and measurements made in the field with conventional exploration methods. Seed and Idriss (1971) first published the widely used “simplified procedure” utilizing the Standard Penetration Test (SPT). Since then, the procedure has evolved, primarily through summary papers by Professor H.B. Seed and his colleagues, and field test methods in addition to the SPT have been utilized in similar simplified procedures. These methods include cone penetrometer tests (CPTs), Becker hammer tests (BHTs), and shear wave velocity tests (SVTs). In 1996, a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T.L. Youd and I.M. Idriss with 20 experts to review and update the simplified procedure which had last been updated in 1985. The update of the simplified procedure that resulted from the NCEER workshop (termed herein the “Liquefaction Workshop”) is summarized in NCEER (1997) and in Youd et al. (2001). Martin and Lew (1999) focused on the implementation of this procedure in engineering practice, especially for southern California. The procedure described in NCEER (1997), Youd et al. (2001), and Martin and Lew (1999) using the Standard Penetration Test (SPT) is later summarized in this section.
2. **Analytical Methods**—Analytical methods are used less frequently to evaluate liquefaction potential – though they may be required for special projects or where soil conditions are not amenable to the empirical method. Analytical methods will also likely gain prominence with time as numerical methods and soil models improve and are increasingly validated. Originally (circa 1970s) the analytical method involved determination of the induced shearing stresses with a program such as SHAKE and comparing these stresses to results of cyclic triaxial or cyclic simple shear tests. Now the analytical method usually refers to a computer code that incorporates a soil model that calculates the buildup in pore water pressure. These more rigorous numerical methods include one-dimensional, nonlinear effective stress codes such as DESRA and SUMDES and two dimensional, nonlinear effective stress codes such as FLAC, TARA, and DYNAFLOW. This new generation of analytical methods has soil models that are fit to laboratory data or liquefaction curves derived from SPT information. The methods are limited by the ability to represent the soil model from either the laboratory or field measurements and by the complexity of the wave propagation mechanisms, including the ability to select appropriate earthquake records to use in the analyses.
3. **Physical Modeling**—These methods typically involved the use of centrifuges or relatively small-scale shaking tables to simulate seismic loading under well defined boundary conditions. More recently these methods have been expanded to include large laminar boxes mounted on very large

shake tables and full-scale field blast loading tests. Physical modeling of liquefaction is one of the main focus areas of the 2004-2014 Network for Earthquake Engineering Simulation (NEES) supported by the National Science Foundation. Soil used in the small-scale and laminar box models is reconstituted to represent different density and geometrical conditions. Because of difficulties in precisely modeling in-situ conditions at liquefiable sites, small-scale and laminar box models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well defined boundary conditions. Recently, blast loading tests have been conducted to capture the in situ characteristics of the soil for research purposes (e.g., Treasure Island, California and in Japan). However, the cost and safety issues of this approach limits its use to only special design or research projects.

The empirical approach for evaluating liquefaction hazards based on the Liquefaction Workshop and described in NCEER (1997) and Youd et al. (2001) is summarized in the following paragraphs.

The first step in the liquefaction hazard evaluation using the empirical approach is usually to define the normalized cyclic shear stress ratio (CSR) from the peak horizontal ground acceleration expected at the site. This evaluation is made using the following simple equation:

$$CSR = 0.65 \left( a_{max}/g \right) \left( \sigma_o / \sigma'_o \right) r_d \quad (C7.4-1)$$

where  $(a_{max}/g)$  = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity,  $\sigma_o$  = the vertical total stress in the soil at the depth in question,  $\sigma'_o$  = the vertical effective stress at the same depth, and  $r_d$  = deformation-related stress reduction factor.

The peak ground acceleration,  $a_{max}$ , commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the  $a_{max}$  used in Eq. C7.4-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop. The acceleration can be determined using the general procedure described in Sec. 3.3 and taking  $a_{max}$  equal to  $S_{DS}/2.5$ . Alternatively,  $a_{max}$  can be estimated from: (1) values obtained from the USGS national ground motion maps [see internet website <http://geohazards.cr.usgs.gov/eq/>] for a selected probability of exceedance, with correction for site effects using the  $F_a$  site factor in Sec. 3.3; or (2) from a site-specific ground motion analysis conforming to the requirements of Sec. 3.4.

The stress reduction factor,  $r_d$ , used in Eq. C7.4-1 was originally determined using a plot developed by Seed and Idriss (1971) showing the reduction factor versus depth. The consensus from the Liquefaction Workshop was to represent  $r_d$  by the following equations:

$$r_d = 1.0 - 0.00765z \text{ for } z \leq 9.15 \text{ m} \quad (C7.4-2)$$

$$r_d = 1.174 - 0.267z \text{ for } 9.15 \text{ m} < z \leq 23 \text{ m}$$

It should be noted that because nearly all the field data used to develop the simplified procedure are for depths less than 12 m, there is greater uncertainty in the evaluations at greater depths. The second step in the liquefaction hazard evaluation using the empirical approach usually involves determination of the normalized cyclic resistance ratio (CRR). The most commonly used empirical relationship for determining CRR was originally compiled by Seed et al. (1985). This relationship compares CRR with corrected Standard Penetration Test (SPT) resistance,  $(N_1)_{60}$ , from sites where liquefaction did or did not develop during past earthquakes. Figure C7.4-3 shows this relationship for Magnitude 7.5 earthquakes, with an adjustment at low values of CRR recommended by the Liquefaction Workshop. Similar relationships have been developed for determining CRR from CPT soundings, from BHT blowcounts, and from shear wave velocity data, as discussed by Youd et al. (2001) and as presented in detail in NCEER (1997). Only the SPT method is presented herein because of its more common use.

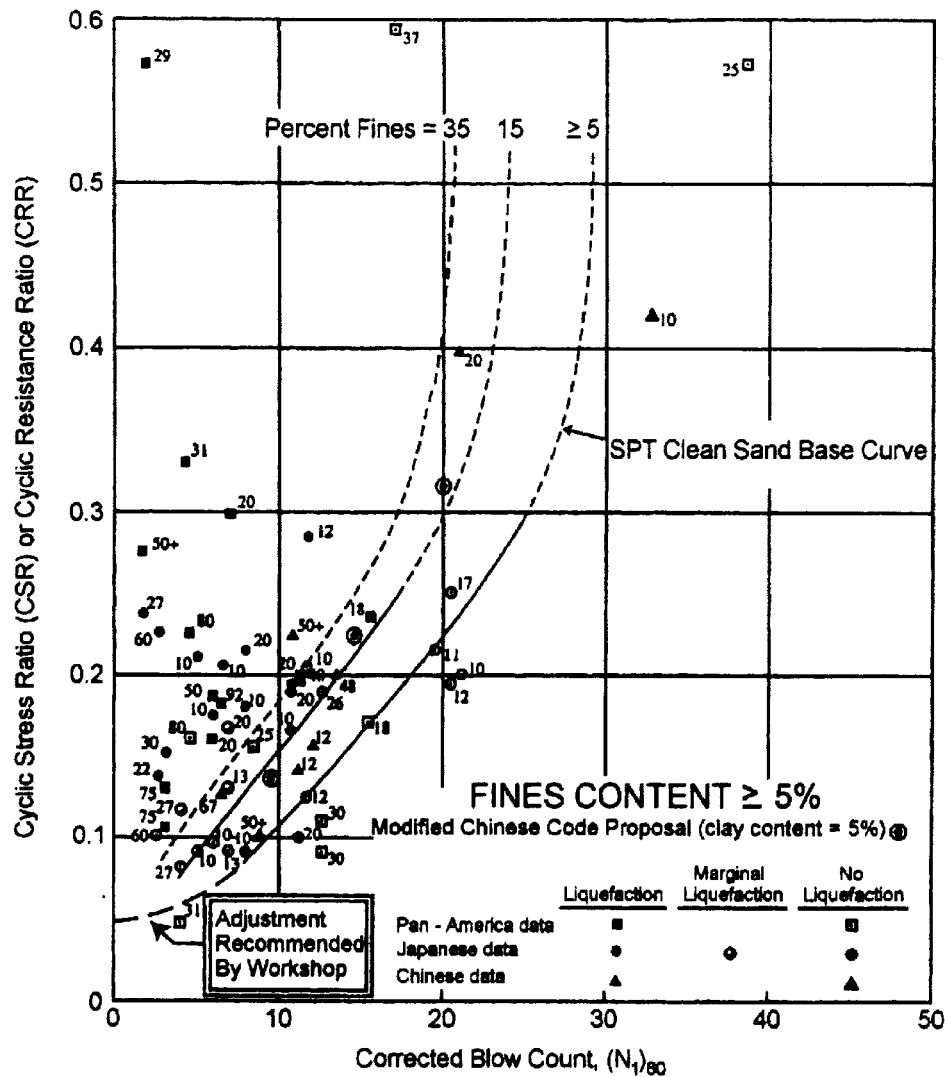


Figure C7.4-3. SPT clean sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories. (Modified from Seed et al., 1985). (NCEER, 1997; Youd et al., 2001).

In Figure C7.4-3, CRRs calculated for various sites are plotted against  $(N_1)_{60}$ , where  $(N_1)_{60}$  is the SPT blowcount normalized for an overburden stress of 100 kPa and for an energy ratio of 60 percent. Solid symbols represent sites where liquefaction occurred and open symbols represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. As shown, curves are given for soils with fines contents (FC) ranging from less than 5 to 35 percent.

While Figure C7.4-3 provides information about the variation in CRR with fines content, the preferred approach from the Liquefaction Workshop for adjusting for fines is to correct  $(N_1)_{60}$  to an equivalent clean sand value,  $(N_1)_{60cs}$  using the following equations:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{C7.4-3}$$

where  $\alpha$  and  $\beta$  = coefficients determined from the following relationships:

- $\alpha = 0$  for  $FC \leq 5\%$
- $\alpha = \exp[1.76 - (190/FC^2)]$  for  $5\% < FC < 35\%$
- $\alpha = 5.0$  for  $FC \geq 35\%$

$$\beta = 1.0 \text{ for FC} \leq 5\%$$

$$\beta = [0.99 + (\text{FC}^{1.5}/1,000)] \text{ for } 5\% < \text{FC} < 35\%$$

$$\beta = 1.2 \text{ for FC} \geq 35\%$$

Several other corrections are made to  $(N_I)_{60}$ , as represented in the following equation:

$$(N_I)_{60} = N_m C_N C_E C_B C_R C_S \quad (\text{C7.4-4})$$

where  $N_m$  = measured standard penetration resistance;  $C_N$  = factor to normalize  $N_m$  to a common reference effective overburden stress;  $C_E$  = correction for hammer energy ratio (ER);  $C_B$  = correction factor for borehole diameter;  $C_R$  = correction factor for rod length; and  $C_S$  = correction for samples with or without liners. Values given in Youd, et al., 2001) are shown in Table C.4-1. An alternative equation for  $C_n$  from that shown in the table (Youd, et al., 2001):

$$C_N = 2.2 / [1.2 + \sigma'_{vo}/Pa] \quad (\text{C7.4-5})$$

where the maximum value of  $C_N$  is equal to 1.7. The effective vertical stress,  $\sigma'_{vo}$ , is the stress at the time of the SPT measurement. Youd et al. (2001) caution that other means should be used to evaluate  $C_N$  if  $\sigma'_{vo}$  is greater than 300 kPa.

Factor	Equipment variable	Term	Correction
Overburden pressure	---	$C_N$	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	---	$C_N$	$C_N < 1.7$
Energy ratio	Donut hammer	$C_E$	0.5 – 1.0
Energy ratio	Safety hammer	$C_E$	0.7 – 1.2
Energy ratio	Automatic-trip Donut-type hammer	$C_E$	0.8 – 1.3
Borehole diameter	65 – 115 mm	$C_B$	1.0
Borehole diameter	150 mm	$C_B$	1.05
Borehole diameter	200 mm	$C_B$	1.15
Rod length	< 3	$C_R$	0.75
Rod length	3 – 4 m	$C_R$	0.8
Rod length	4 -6 m	$C_R$	0.85
Rod length	6 – 10 m	$C_R$	0.95
Rod length	10 – 30 m	$C_R$	1.0
Sampling method	Standard sampler	$C_S$	1.0
Sampling method	Sampler without liners	$C_S$	1.1 – 1.3

It is very important that the engineer consider these correction factors when conducting the liquefaction analyses. Failure to consider these corrections can result in inaccurate liquefaction estimates – leading

to either excessive cost to mitigate the liquefaction concern or excessive risk of poor performance during a design event – potentially resulting in unacceptable damage.

Special mention also needs to be made of the energy calibration term,  $C_E$ . This correction has a very significant effect on the  $(N_1)_{60}$  used to compute CRR. The value of this correction factor can vary greatly depending on the SPT hammer system used in the field and on site conditions. For important sites where  $C_E$  could result in changes from liquefied to non liquefied, energy ratio measurements should be made. These measurements are relatively inexpensive and represent a small increase in overall field exploration costs. Many drilling contractors in areas that are seismically active provide calibrated equipment as part of their routine service.

Before computing the factor of safety from liquefaction, the CRR result obtained from Figure C7.4-3 (using the corrected SPT blow count identified in the equation for  $(N_1)_{60}$ ) must be corrected for earthquake magnitude  $M$  if the magnitude differs from 7.5. The magnitude correction factor is shown in Figure C7.4-4. This plot was developed during the Liquefaction Workshop on the basis of input from experts attending the workshop. The range shown in Figure C7.4-4 is used because of uncertainties. The user should select a value consistent with the project risk. For  $M$  greater than 7.5 the factors recommended by Idriss (second from highest) should be used.

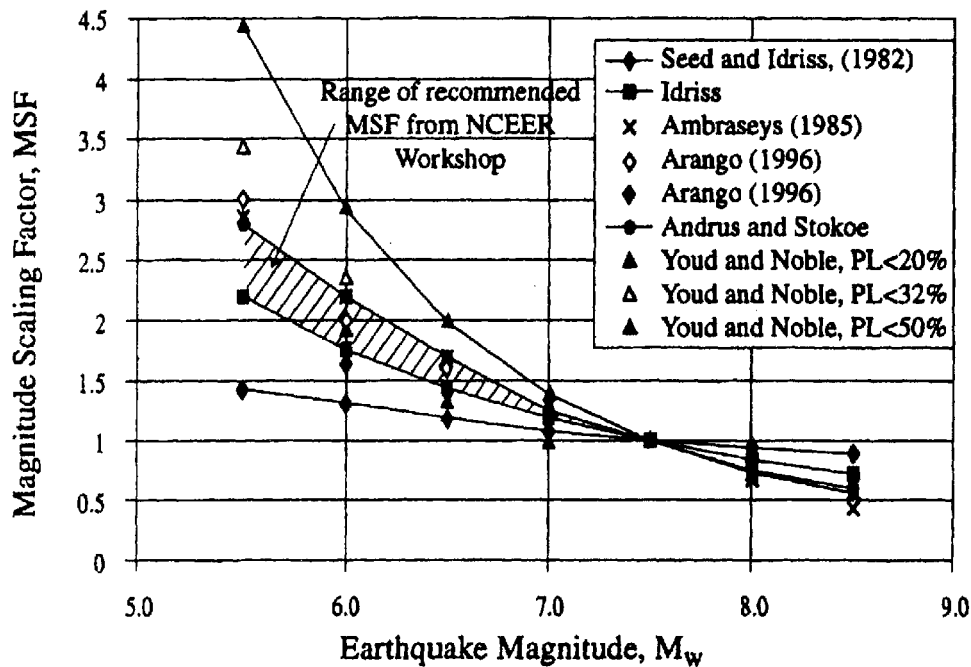


Figure C7.4-4. Magnitude scaling factors derived by various investigators. (NCEER, 1997; Youd et al., 2001)

The magnitude,  $M$ , needed to determine a magnitude scaling factor from Figure C7.4-4 should correspond to the Maximum Considered Earthquake (MCE). Where the general procedure for ground motion estimation is used (Sec. 3.3) and the MCE is determined probabilistically, the magnitude used in these evaluations can be obtained from deaggregation information available by latitude and longitude from the USGS website (<http://geohazards.cr.usgs.gov/eq/>). Where the general procedure (Sec. 3.3) is used and the MCE is bounded deterministically near known active fault sources (*Commentary* Appendix A), the magnitude of the MCE should be the characteristic maximum magnitude assigned to the fault in the construction of the MCE ground motion maps. Where the site-specific procedure for ground motion estimation is used (Sec. 3.4), the magnitude of the MCE should be similarly determined from the site-specific analysis. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety  $F_L$  in Eq. C7.4-6) is determined jointly by  $a_{max}$  and  $M$  and not by  $a_{max}$  alone. Because of the longer duration of strong ground-shaking, large distant earthquakes may in

some cases generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though  $a_{max}$  of the nearer events is larger than that from the more distant events.

The final step in the liquefaction hazard evaluation using the empirical approach involves the computation of the factor of safety ( $F_L$ ) against liquefaction using the equation:

$$F_L = \text{CRR}/\text{CSR} \quad (\text{C7.4-6})$$

If  $F_L$  is greater than one, then liquefaction should not develop. If at any depth in the sediment profile,  $F_L$  is equal to or less than one, then there is a liquefaction hazard. Although the curves shown in Figure C7.4-3 envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. For this reason a factor of safety of 1.2 to 1.5 is usually appropriate for building sites – with the actual factor selected on the basis of the importance of the structure and the potential for ground displacement at the site.

Additional guidance on the selection of the appropriate factor of safety is provided by Martin and Lew 1999. They suggest that the following factors be considered when selecting the factor of safety:

1. The type of structure and its vulnerability to damage.
2. Levels of risk accepted by the owner or governmental regulations with questions related to design for life safety, limited structural damage, or essentially no damage.
3. Damage potential associated with the particular liquefaction hazards. Flow failures or major lateral spreads pose more damage potential than differential settlement. Hence factors of safety could be adjusted accordingly.
4. Damage potential associated with design earthquake magnitude. A magnitude 7.5 event is potentially more damaging than a 6.5 event.
5. Damage potential associated with SPT values; low blow counts have a greater cyclic strain potential than higher blowcounts.
6. Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15 percent could change a factor of safety from, say, 1.0 to 1.25.
7. For high levels of design ground motion, factors of safety may be indeterminate. For example, if  $(N_1)_{60} = 20$ ,  $M = 7.5$ , and fines content = 35 percent, liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

Martin and Lew (1999) indicate that the final choice of an appropriate factor of safety must reflect the particular conditions associated with the specific site and the vulnerability of site-related structures. Table C7.4-2 summarizes factors of safety suggested by Martin and Lew.

**Table C7.4-2. Factors of safety for liquefaction hazard assessment (from Martin and Lew, 1999).**

Consequences of Liquefaction	$(N_1)_{60cs}$	Factor of Safety
Settlement	$\leq 15$	1.1
	$\geq 30$	1.0
Surface Manifestations	$\leq 15$	1.2
	$\geq 30$	1.0
Lateral Spread	$\leq 15$	1.3
	$\geq 30$	1.0

As a final comment on the assessment of liquefaction hazards, it is important to note that soils composed of sands, silts, and gravels are most susceptible to liquefaction while clayey soils generally are not susceptible to this phenomenon. The curves in Figure C7.4-3 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered. For soils containing more than 35 percent fines, the curve in Figure C7.4-3 for 35 percent fines should be used provided the following criteria are met (Seed and Idriss, 1982; Seed et al., 1983): the weight of soil particles finer than 0.005 mm is less than 15 percent of the dry weight of a specimen of the soil; the liquid limit of soil is less than 35 percent; and the moisture content of the in-place soil is greater than 0.9 times the liquid limit. If these criteria are not met, the soils may be considered nonliquefiable.

**Evaluation of potential for ground displacements.** Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support and/or ground deformation does this phenomenon become important to structural design. Surface manifestations, loss of bearing strength, ground settlement, flow failure and lateral spread are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described in Martin and Lew (1999), U.S. Army Corps of Engineers (1998) and National Research Council (1985) and are discussed below. The type of failure and amount of ground displacement are a function of several parameters including the looseness of the liquefied soil layer, the thickness and extent of the liquefied layer, the thickness and permeability of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face.

*Surface Manifestations.* Surface manifestations refer to sand boils and ground fissures on level ground sites. For structures supported on shallow foundations, the effects of surface manifestations on the structure could be tilting or cracking. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects for level sites. These criteria may be used for noncritical or nonessential structures on level sites not subject to lateral spreads (see later in this section). Additional analysis should be performed for critical or essential structures.

*Loss of bearing strength.* Loss of bearing strength can occur if the foundation is located within or above the liquefiable layer. The consequence of bearing failure could be settlement or tilting of the structure. Usually, loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. Simple guidance for how deep or how thin the layers must be has not yet been developed. Martin and Lew (1999) provide some preliminary guidance based on the Ishihara (1985) method. Final evaluation of the potential for loss of bearing strength should be made by a geotechnical engineer experienced in liquefaction hazard assessment

*Ground settlement.* For saturated or dry granular soils in a loose condition, the amount of ground settlement could approach 3 to 4 percent of the thickness of the loose soil layer in some cases. This amount of settlement could cause tilting or cracking of a building, and therefore, it is usually important to evaluate the potential for ground settlement during earthquakes.

Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this commentary to outline that procedure which, although explicit, has several rather complex steps. The Tokimatsu and Seed procedure can be applied whether liquefaction does or does not occur. For dry cohesionless soils, the settlement estimate from Tokimatsu and Seed should be multiplied by a factor of 2 to account for multi-directional shaking effects as discussed by Martin and Lew (1999).

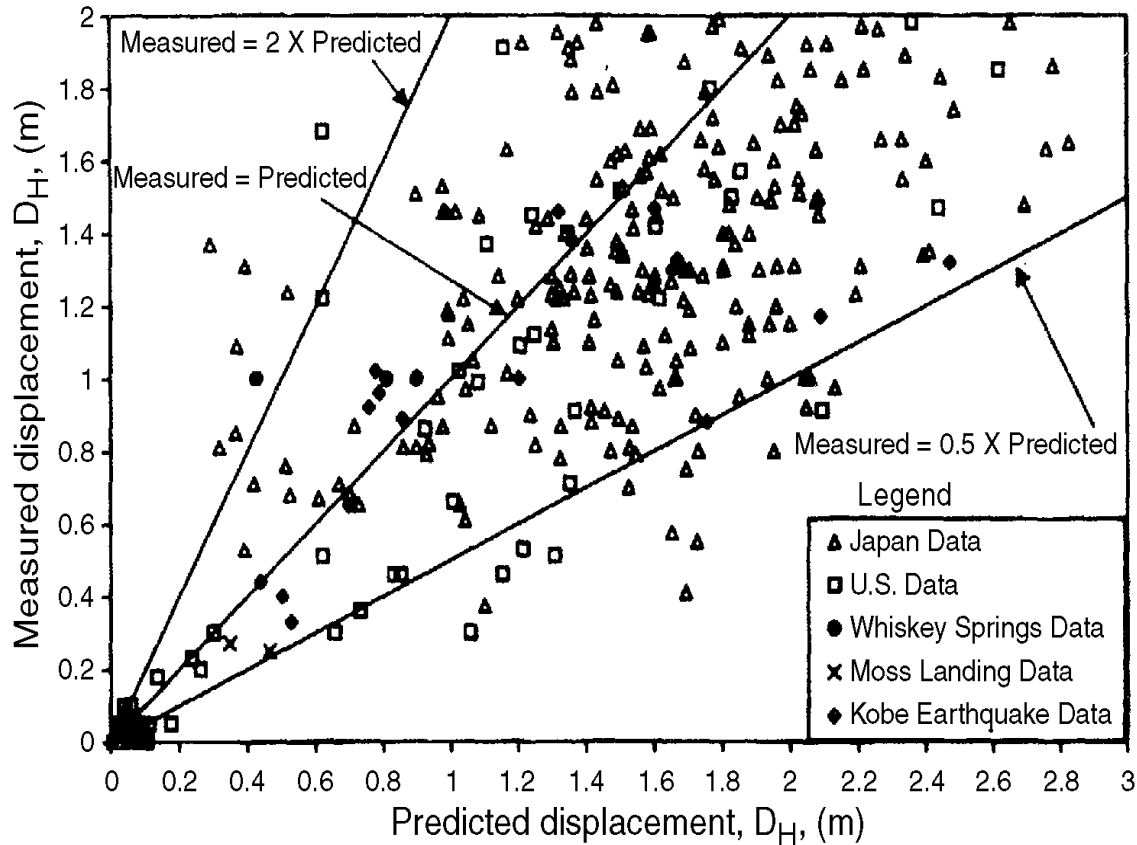
*Flow failures.* Flow failures or flow slides are the most catastrophic form of ground failure that may be triggered when liquefaction occurs. They may displace large masses of soils tens of meters. Flow slides occur when the average static shear stresses on potential failure surfaces are less than the average shear strengths of liquefied soil on these surfaces. Standard limit equilibrium static slope stability analyses may be used to assess flow failure potential with the residual strength of liquefied soil used as the strength parameter in the analyses.

The determination of residual strengths is very inexact, and consensus as to the most appropriate approach has not been reached to date. Two relationships for residual strength of liquefied soil that are often used in practice are those of Seed and Harder (1990) and Stark and Mesri (1992). A more complete discussion and references on this topic may be found in Martin and Lew (1999).

*Lateral spreads.* Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. They may result in lateral movements in the range of a few centimeters to several meters. Earthquake ground-shaking affects the stability of gently sloping ground containing liquefiable materials by seismic inertia forces combined with static gravity forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces are manifested by lateral “downslope” movement. For the duration of ground shaking associated with moderate-to large-magnitude earthquakes, there could be many such occurrences of temporary instability during earthquake shaking, producing an accumulation of “downslope” movement.

Various analytical and empirical techniques have been developed to date to estimate lateral spread ground displacement; however, no single technique has been widely accepted or verified for engineering design. Three approaches are used depending on the requirements of the project: empirical procedures, simplified analytical methods, and more rigorous computer modeling. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. Youd et al. (2002) present an empirical method that provides an estimate of lateral spread displacements as a function of earthquake magnitude, distance, topographic conditions, and soil deposit characteristics. As shown in Figure C7.4-5, the displacements estimated by the Youd et al. (2002) method are generally within a factor of two of the observed displacements. Bardet et al. (2002) present an empirical method having a formulation similar to that of Youd et al. (2002) but using fewer parameters to describe the soil deposit. The Bardet et al. (2002) model was developed to assess lateral spread displacements at a regional scale rather than for site-specific applications.

Simplified analytical techniques generally apply some form of Newmark’s analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the residual strength of the deforming soil. Additional discussion of the simplified Newmark method is provided in Sec. 7.4.1. More rigorous computer modeling typically involves use of nonlinear finite element or finite difference methods to predict deformations, such as with the computer code FLAC. Both the simplified Newmark method and the rigorous computer codes require considerable experience to obtain meaningful results. For example, the soil model within the nonlinear computer codes is often calibrated for only specific conditions. If the site is not characterized by these conditions, errors in estimating the displacement by a factor of two or more can easily occur.



**Figure C7.4-5. Measured versus predicted displacements for displacements up to 2 meters. (Youd et al., 2002).**

Liquefaction-induced deformations are not directly proportional to ground motions and may be more than 50 percent higher for maximum considered earthquake ground motions than for design earthquake ground motions. The liquefaction potential and resulting deformations for ground motions consistent with the maximum considered earthquake should also be evaluated and, while not required in the *Provisions*, should be used by the registered design professional in checking for building damage that may result in collapse. In addition, Seismic Use Group III structures should be checked for their required post-earthquake condition.

**Mitigation of liquefaction hazard.** With respect to the hazard of liquefaction, three mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties as discussed in Sec. 7.4.3. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, may receive reduced soil support through the liquefied layer and may be subjected to transient lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 ft, although re-leveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a foot.

Evaluations of structural performance following two recent Japanese earthquakes, 1993 Hokkaido Nansei-Oki and 1995 (Kobe) Hyogo-Ken Nanbu, indicate that small structures on shallow foundations performed well in liquefaction areas. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to several tenths of a meter of lateral spread displacement. Many small structures (mostly houses, shops, schools, etc.) were

structurally undamaged although a few tilted slightly. Foundations for these structures consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

Similarly, well reinforced foundations that would not fracture could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral ( $< 0.3$  m) or vertical ( $< 0.05$  m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural components.

Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls as discussed in Sec. 7.5.1. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further explanation of these methods is given by the National Research Council (1985).

**Surface fault rupture hazard.** Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, generally occur along traces of previously active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these components. The following commentary summarizes procedures to follow or consider when assessing the hazard of surface fault rupture. Sources of detailed information for evaluating the hazard of surface fault rupture include Slemmons and dePolo (1986), the Utah Section of the Association of Engineering Geologists (1987), Swan et al. (1991), Hart and Bryant (1997), and California Geological Survey (2002). Other beneficial references are given in the bibliographies of these publications.

*Assessment of surface faulting hazard.* The evaluation of surface fault rupture hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing active faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications related to foundation design, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart and Bryant, 1997): “An active fault has had displacement in Holocene time (last 11,000 years).”

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, the amount and character of past displacements, and the expected amount and potential of future displacement. Identification and

characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies often include trenching to accurately locate, document, and date fault features.

*Suggested approach for assessing surface faulting hazard.* The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. However, it is not expected that all of the following methods would be used in a single investigation:

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.
2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography/geomorphic features, vegetation and soil contrasts, and other lineaments of possible fault origin. The study of predevelopment aerial photographs is often essential to the detection of fault features.
3. A field reconnaissance study generally is required and should include observation and mapping of bedrock and soil units and structures, geomorphic surfaces, fault-related geomorphic features, springs, and deformation of man-made structures due to fault creep. Field study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site.
4. Subsurface investigations usually are needed to evaluate location and activity of fault traces. These investigations may include trenches, test pits, and/or boreholes to permit detailed and direct observation of geologic units and faults.
5. The geometry of faults may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require a knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone never prove the absence of a fault and they typically do not identify the recency of activity.
6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis ( $C^{14}$ , K-Ar), stratigraphic correlation (fossils, mineralogy) soil profile development, paleomagnetism (magnetostratigraphy), or other dating techniques (thermoluminescence, cosmogenic isotopes) to date the age of faulted or unfaulted units or surfaces. Probabilistic studies may be considered to evaluate the probability of fault displacement (Youngs et al., 2003).

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of hazardous faults on or near the site. The distribution of primary and secondary faulting (fault zone width) and fault-related surface deformation should be shown.
2. The type, amount, and sense of displacement of past surface faulting episodes should be documented, if possible.
3. From this documentation, estimates of location, magnitude, and likelihood or relative potential for future fault displacement can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from empirical correlations between fault displacement and fault length or earthquake magnitude published by Wells and Coppersmith (1994). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected).
4. The degree of confidence and limitations of the data should be addressed.

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists using techniques described above. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimates reports, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, the general setback requirement in California is a minimum of 50 ft from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safely applied (*California Code of Regulations*, Title 14, Division 2, Sec. 3603(a)).
2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved. Where sufficient geologic data have been developed to accurately locate the zone containing active fault traces and the zone is not complex, a 50-ft setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

*Mitigation of surface faulting hazards.* There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture. In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989; Kelson et al., 2001). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

**7.4.2 Pole-type structures.** The use of pole-type structures is permitted. These structures are inherently sensitive to earthquake motions. Bending in the poles and the soil capacity for lateral resistance of the portion of the pole embedded in the ground should be considered and the design completed accordingly.

**7.4.3 Foundation ties.** One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation that acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to  $S_{DS}/10$  times the larger pile cap or column load.

A common practice in some multistory buildings is to have major columns that run the full height of the building adjacent to smaller columns in the basement that support only the first floor slab. The coefficient applies to the heaviest column load.

Alternate methods of tying foundations together are permitted (such as using a properly reinforced floor slab that can take both tension and compression). Lateral soil pressure on pile caps is not a recommended method because the motion is imparted from soil to structure (not inversely as is commonly assumed), and if the soil is soft enough to require piles, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions.

If piles are to support structures in the air or over water (such as in a wharf or pier), batter piles may be required to provide stability or the piles may be required to provide bending capacity for lateral stability. It is up to the foundation engineer to determine the fluidity or viscosity of the soil and the point where lateral buckling support to the pile can be provided (that is, the point where the flow of the soil around the piles may be negligible).

**7.4.4 Special pile requirements.** Special requirements for piles, piers, or caissons in Seismic Design Category C are given in this section. Provisions for pile anchorage to the pile cap or grade beam and transverse reinforcement detailing requirements for concrete piles are provided. The anchorage requirements are intended to assure that the connection to the pile cap does not fail in a brittle manner under moderate ground motions. Moderate ground motions could result in pile tension forces or bending moments which could compromise shallow anchorage embedment. Shallow anchorages in pile caps may consist of short lengths of reinforcing bars or bare structural steel pile sections. Loss of pile anchorage could result in unintended increases in vertical seismic force resisting element drifts from rocking, potential overturning instability of the superstructure, and loss of shearing resistance at the ground surface. Anchorage by shallow embedment of the bare steel pile section is not recommended due to the degradation of the concrete bond from cracking as a result of the cyclic loading from the moderate ground motions. Exception to this is permitted for steel pipe piles filled with concrete when the connection is made with reinforcing bar dowels properly developed into the pile and pile cap. The confinement of the interior concrete by the “hoop” stresses of the circular pile section was judged to be sufficient to prevent concrete pullout from that section. Using this method of connection, the structural steel pipe section should be embedded into the pile cap for a short distance or else the pile should be designed as an uncased concrete pile. End anchorage detailing requirements for transverse reinforcement generally follow that required by ACI 318, Chapter 21 to assure that no loss of confinement of the transverse reinforcement occurs in concrete piles since verification of pile damage after moderate ground motions is difficult or not done.

**7.4.4.1 Uncased concrete piles.** The uncased concrete piles category has been expanded to include auger-cast piles which are now subject to the same reinforcement requirements as other cast-in-place concrete piles. The longitudinal reinforcement within a pile has prescriptive termination point minimums intended for firm soils and is required to extend at least the full flexural length. The longitudinal reinforcement should extend past the flexural length by its development length requirement. The flexural length has been defined as that length of a pile from bottom of pile cap to a point where 0.4 times the concrete section cracking moment exceeds the calculated flexural demand. The 0.4 factor is analogous to a material resistance factor in strength design. This definition implies the plain concrete section will be resisting some minimal amount of moment demand beyond the “flexural” length. Where the pile is subject to significant uplift forces, it is recommended that the longitudinal reinforcement extend the full length of the pile.

Increased transverse reinforcement requirements are given for the potential plastic hinge zone immediately below the pile cap. The potential plastic hinge zone was taken to be three pile diameters below the pile cap to allow for varied soil site classes. The transverse reinforcement detailing for this zone is similar to that required for concrete intermediate moment frames at hinge regions and is expected to provide a displacement ductility of approximately 4. Beyond the potential plastic hinge region, the curvature ductility demand is not considered to exceed that provided by the nominal moment capacity of the section for non-earthquake loads.

**7.4.4.4 Precast (non-prestressed) concrete piles.** For precast concrete piles, the longitudinal reinforcement is specified to extend the full length of the pile so there is no need to determine the flexural length. Transverse reinforcement spacing within the potential plastic hinge zone is required for the length of three pile diameters at the bottom of pile cap. Particular attention should be taken where piles cannot be driven to or are overdriven beyond the anticipated end bearing point elevation. The transverse reinforcement size and spacing in this region is the same as the uncased concrete pile. Transverse reinforcement spacing outside the potential plastic hinge zone is specified to be no greater than 8 inches to conform with current building code minimums for this pile type.

**7.4.4.5 Precast-prestressed piles.** The transverse reinforcement requirements are primarily taken from the PCI Committee Report (1993) on precast prestressed concrete piling for geographic regions subject to low to moderate ground motions. The amount of transverse reinforcement was relaxed for the pile region greater than 20 feet (6m) below the pile cap to one-half of that required above. It was judged that the reduced transverse reinforcement would be sufficient to resist the reduced curvature demands at that point. Particular attention should be taken where piles cannot be driven to or are overdriven beyond the anticipated end bearing point elevation so that the length of the confining transverse reinforcement is maintained.

Equation (7.4-1), originally from ACI 318, has always been intended to be a lower bound spiral transverse reinforcement ratio for larger diameter columns. It is independent of the member section properties and can therefore be applied to large or small diameter piles. For cast-in-place piles and prestressed concrete piles, the resulting spiral reinforcement ratios from this formula are considered to be sufficient to provide moderate ductility capacities.

High strength hard drawn wire with higher yield strengths is permitted to be used for transverse circular spiral reinforcement of precast prestressed concrete piles. Pile test specimens using this type of transverse reinforcement include the research done by Park and Hoat Joen (1990). High strength hard drawn wire has yield strengths between 150 and 200 ksi.  $f_{yh}$  is conservatively limited to 85 ksi for this steel because hard drawn wire has limited ductility.

## 7.5 SEISMIC DESIGN CATEGORIES D, E, AND F

For Seismic Design Category D, E, or F construction, all the preceding provisions for Seismic Design Category C applies for the foundations, but the earthquake detailing is generally more severe and demanding.

**7.5.1 Investigation.** In addition to the potential site hazards discussed in *Provisions* Sec. 7.4.1, consideration of lateral pressures on earth retaining structures shall be included in investigations for Seismic Design Categories D, E, and F.

**Earth retaining structures.** Increased lateral pressures on retaining structures during earthquakes have long been recognized; however, design procedures have not been prescribed in U.S. model building codes. Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. Damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that “engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior.”

Seismic design analysis of retaining walls is discussed below for two categories of walls: “yielding” walls that can move sufficiently to develop minimum active earth pressures and “nonyielding” walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever

walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

*Yielding walls.* At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Monobe-Okabe (M-O) seismic coefficient analysis (Monobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1 - k_v) K_{AE} \quad (C7.5-1)$$

where:  $P_{AE}$  is the total (static + dynamic) lateral thrust,  $\gamma$  is unit weight of backfill soil,  $H$  is height of backfill behind the wall,  $k_v$  is vertical ground acceleration divided by gravitational acceleration, and  $K_{AE}$  is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for  $K_{AE}$  is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust,  $P_{AE}$ , in terms of its static component ( $P_A$ ) and dynamic incremental component ( $\Delta P_{AE}$ ):

$$P_{AE} = P_A + \Delta P_{AE} \quad (C7.5-2a)$$

or

$$K_{AE} = K_A + \Delta K_{AE} \quad (C7.5-2b)$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \quad (C7.5-2c)$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} = (3/4)K_h \quad (C7.5-3a)$$

$$\Delta P_{AE} = (1/2)\gamma H^2 (3/4)k_h = (3/8)k_h \gamma H^2 \quad (C7.5-3b)$$

where  $k_h$  is horizontal ground acceleration divided by gravitational acceleration. It is recommended that  $k_h$  be taken equal to the site peak ground acceleration that is consistent with design earthquake ground motions as determined in *Provisions* Sec. 7.5.2 (that is,  $k_h = S_{DS}/2.5$ ). Eq. C7.5-3a and C7.5-3b generally are referred to as the simplified M-O formulation.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust,  $\Delta P_{AE}$ , Seed and Whitman (1970) recommended that the resultant dynamic thrust act at  $0.6H$  above the base of the wall (that is, inverted trapezoidal pressure distribution).

Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. There are a number of empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by

Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash and others (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards and others (1996).

Application of methods for evaluating tilting of yielding walls has been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgment. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation in order to displace only in the sliding mode.

*Nonyielding walls.* Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (that is, motions at less than half of the natural frequency of the unconstrained backfill), which would include many or most earthquake problems.

For uniform, constant  $k_h$  applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust,  $\Delta P_E$ , acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \quad (\text{C7.5-4a})$$

The value of  $F$  is approximately equal to unity (Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \quad (\text{C7.5-4b})$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of  $0.6H$  above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base. The subject of soil-wall interaction is addressed in the following sections. This section also provides further discussion on the applicability of the Wood and the M-O formulations.

**Soil-structure-interaction approach and modeling for wall pressures.** Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan and White (1998), among others, argue that the earth pressures acting on the walls of embedded structures during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (that is, M-O method). Soil-structure interaction includes both a kinematic component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973)—and an inertial component—the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et al., 1975) and SASSI (Lysmer et al., 1981).

Ostadan and White (1998) have observed that for embedded structures subjected to ground shaking, the characteristics of the wall pressure amplitudes vs. frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude

and distribution of dynamic thrust. Results provided by Ostadan and White (1998) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a “lower” and an “upper” bound, respectively.

Chang and others (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang and others indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure. Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (that is, no significant rocking). In the latter case, the pressures given by the Wood (1973) formulation would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial components are out of phase.

*Effect of saturated backfill on wall pressures.* The previous discussions are limited to backfills that are not water-saturated. In current (1999) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (such as quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992).

A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present (1999), there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these “static” fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).

**7.5.3 Foundation ties.** The additional requirement is made that spread footings on soft soil profiles should be interconnected by ties. The reasoning explained above under Sec. 7.4.3 also applies here.

**7.5.4 Special pile and grade beam requirements.** For Seismic Design Categories D, E, and F, additional pile reinforcement over that specified for Seismic Design Category C buildings is required. Adequate pile ductility is required and provision must be made for additional reinforcement to ensure, as a minimum, full ductility in the upper portion of the pile.

Special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to pile design where portions of the foundation piles are supported in soils such as loose granular materials and/or soft soils that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking, for example:

1. Soil settlement at the pile-cap interface either from consolidation of soft soil prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.
2. Large deformations and/or reduction in strength resulting from liquefaction of loose granular materials can cause bending and/or conditions of free-standing columns.
3. Large deformations in soft soils can cause varying degrees of pile bending. The degree of pile bending will depend upon thickness and strength of the soft soil layer(s) and/or the properties of the soft/stiff soil interface(s).

Such conditions can produce shears and/or curvatures in piles that may exceed the bending capacity of conventionally designed piles and result in severe damage. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and Mylonakis (2001) and these effects on concrete piles are further discussed by Shepard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a pile present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil, although considerable judgment is necessary in utilizing this simple relationship in a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction with regard to pile foundation behavior.

The designer needs to consider the variation in soil conditions and driven pile lengths in providing for pile ductility at potential high curvature interfaces. Interaction between the geotechnical and structural engineers is essential.

It is prudent to design piles to remain functional during and following earthquakes in view of the fact that it is difficult to repair foundation damage. The desired foundation performance can be accomplished by proper selection and detailing of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves. Examples of designs of concrete piles include:

1. Use of a heavy spiral reinforcement and
2. Use of exterior steel liners to confine the concrete in the zones with large curvatures or shear stresses.

These provide proper confinement to ensure adequate ductility and maintenance of functionality of the confined core of the pile during and after the earthquake.

Design of piles incorporates the same  $R$  force reduction factor as the superstructure and therefore implies inelasticity in the foundations and piles. Therefore, piles should be designed with similar ductility requirements as the superstructure. Foundations in SDC D, E, and F are expected to experience strong ground motions and large pile curvatures. Inertial pile-soil-structure interaction may produce plastic hinging in the piles near the bottom of the pile cap. Kinematic soil-pile-structure interaction will result in some bending moments and shearing forces throughout the length of the pile and will be higher at interfaces between stiff and soft soil strata. Inertial pile-soil-structure interaction will be particularly severe in soft soils and liquefiable soils located near the pile cap. This could result in plastic hinging of the pile in reverse curvature near the pile cap and for this reason the potential plastic hinge region is extended to seven pile diameters from the pile cap in the *Provisions*.

Precast prestressed concrete piles are exempted from the concrete special moment frame detailing requirements adapted for concrete piles since these provisions were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. Piles with substantially less confinement reinforcement than required by ACI 318 equation 10-6 have been proven through cyclic testing to have adequate performance (Park and Hoat Joen, 1990). Transverse steel requirements for the precast prestressed concrete piles are given in Section 7.5.4.4.

Where grade beams have the strength to resist the load combination with overstrength, which simulates

expected foundation demands under a yielding structure, detailing similar to the beams of the Special Moment Frame is not required. This “strong grade beam” provision could apply to both cantilever column systems and frame systems with the objective of avoiding the inelastic response or plastic hinging of the grade beam where it would be difficult to detect and repair after being subjected to strong ground motions.

Anchorage of the pile to the pile cap should be designed to permit energy dissipating mechanisms, such as pile slip at the pile-soil-interface, while maintaining a competent connection to the pile cap. A “least” capacity design approach is used for this purpose based on the pile section strength, not to exceed the load combination with overstrength, which simulates expected foundation demands under a yielding structure. A similar approach is also used for pile splice design.

Provisions are given to establish requirements as to when different pile analysis methods should be used. Short piles and long slender piles embedded in the earth behave differently when subject to lateral forces and displacements. Long slender pile response depends on its interaction with the soil considering the non-linear response of the soil. Long slender piles should be analyzed for lateral loads considering the nonlinear interaction of the shaft and soil. The nonlinearity is typically considered in the soil and not the pile. Numerous design aid curves and computer programs are available for this type of analysis, and such an analysis is not uncommon in practice (e.g. Ensoft, 2000a). This type of analysis is necessary to obtain realistic pile moments, forces and deflections. More sophisticated analyses which also consider nonlinear behavior or plastic hinging in the pile itself as well as nonlinearity in the soil for actual earthquake ground motions is still in the research realm. For pile length-to-diameter ratios less than or equal to 6, the pile can be treated as a rigid body simplifying the analysis. A method assuming a rigid body and linear soil response for lateral bearing is in current building codes. A more accurate and comprehensive approach using this method is given in a study by Czerniak (1957).

The effects of groups of piles, where closely spaced, should be taken into account for the soil vertical and horizontal response. As groups of closely spaced piles move laterally, failure zones for individual piles overlap and horizontal strength and stiffness response of the pile-soil system is reduced. Reduction factors or “*p-multipliers*” are needed to account for these groups of closely spaced piles. For a pile center-to-center spacing of three pile diameters, reduction factors of 0.6 for the leading pile row and 0.4 for the trailing pile rows are recommended by Rollins et al. (1999). Computer programs are available to analyze group effects, (e.g. Ensoft, 2000b).

Batter pile systems that are partially embedded have historically performed poorly under strong ground motions (Gerwick and Fotinos, 1992). Failure of battered piles has been attributed to neglecting the potential loading on the piles from ground deformation and also to an erroneous assumption that the lateral loads will be resisted by the axial response of piles leading to neglect of the induced moments in the pile at the pile cap (Lam and Bertero, 1990). Difficulties in examining fully embedded batter piles have led to uncertainties of the extent of damage for this type of foundation. Batter piles are considered as limited ductile systems by their nature and should be designed using the load combination with overstrength. Due to eccentricities inherent in batter pile configurations, moment resisting connections to the pile cap are required to resolve the statics. Otherwise the superstructure will have to resolve the eccentricities by resisting moments induced by the foundation under lateral forces. This concept is clearly illustrated in EQE Engineering (1991).

**7.5.4.1 Uncased concrete piles.** The uncased concrete piles category has been expanded to include auger-cast piles which are now subject to the same reinforcement requirements as other cast-in-place concrete piles. The longitudinal reinforcement within a pile has prescriptive termination point minimums intended for firm soils and is required to extend at least the full flexural length. The longitudinal reinforcement should extend past the flexural length by its development length requirement. The flexural length has been defined as that length of a pile from bottom of pile cap to a point where 0.4 times the concrete section cracking moment exceeds the calculated flexural demand. The 0.4 factor is analogous to a material resistance factor in strength design. This definition implies the plain concrete section will be resisting some minimal amount of moment demand beyond the “flexural” length. Where the pile is subject to significant uplift forces, it is recommended that the longitudinal

reinforcement extend the full length of the pile.

Increased transverse reinforcement requirements are given for the potential plastic hinge zone immediately below the pile cap and for regions beyond that zone. The potential plastic hinge zone was taken to be three pile diameters below the pile cap to allow for the varied soil site classes from A through D. For soil site classes E and F, the potential plastic hinge zone is taken to be seven diameters in length as given in Section 7.5.4. The transverse reinforcement detailing for these zones is similar to that required for concrete Special Moment Frames at hinge regions and is expected to provide a displacement ductility of approximately 5 to 6 depending upon the axial load. However, recent studies and testing has substantiated that the soil will contribute substantially to the confinement of the concrete pile section in firm soils. Chai and Hutchison (1998) found that in-situ lateral load testing of 16 inch diameter conventionally reinforced circular piles performed to displacement ductilities from approximately 3 to 4 using a spiral steel reinforcement ratio of 0.0057. Further testing of the same piles but with a spiral steel reinforcement ratio of 0.0106 produced improved displacement ductilities and no spiral fracture failure which occurred in the prior piles tested with the lower spiral ratio. These circular spiral ratios are considerably less than those required by ACI 318 for columns in Seismic Design Category D. ACI 318 equation 10-6 would require a spiral reinforcement ratio of at least 0.0175 depending on the concrete core diameter. Budek, Benzoni and Priestley (1997) found that testing of 24 inch diameter circular conventionally reinforced concrete piles with a transverse (circular) reinforcement ratio of 0.006 offered adequate performance up to a displacement ductility of 4. Their conclusions indicate that the soil confinement can play a significant role in the pile shaft response. As a result of these studies, full confinement reinforcement intended for superstructure columns is not necessary for in-ground pile foundations, except for soil site classes E and F and liquefiable soils. Beyond the potential plastic hinge region, the curvature ductility demand is not considered to exceed that provided by the nominal moment capacity of the section for nominal earthquake loads.

**7.5.4.4 Precast-prestressed piles.** The transverse reinforcement requirements are primarily taken from the PCI Committee Report (1993) on precast prestressed concrete piling for geographic regions subject to strong ground motions. The circular spiral transverse reinforcement equations recommended for precast prestressed concrete piles given in Park and Hoat Joen (1990) are the basis for the provisions. These equations make the curvature ductility capacity dependent on the pile axial load. A reduction of 50% in the normally required circular spiral reinforcement from those equations (similar to ACI 318 equation 10-6) was sufficient to achieve a displacement ductility of 4. The reduced circular spiral transverse reinforcement requirement is the basis for the PCI Piling Committee's final provisions.

High strength hard drawn wire with higher yield strengths is permitted to be used for transverse circular spiral reinforcement of precast prestressed concrete piles. Pile test specimens using this type of transverse reinforcement include the research done by Park and Hoat Joen (1990). High strength hard drawn wire has yield strengths between 150 and 200 ksi.  $f_{yh}$  is conservatively limited to 85 ksi for this steel because hard drawn wire has limited ductility.

**7.5.4.5 Steel Piles.** AISC Seismic (2002), Part I, Section 8.6 provides seismic design and detailing provisions for steel H-piles which should be used in conjunction with these provisions.

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